EFFECTIVE STRESS METHODS APPLIED TO OFFSHORE PILES

by
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INTRODUCTION

General

The modern use of piles to support offshore structures is more than thirty years old, beginning in the late 1940's development of petroleum in the Gulf of Mexico. During this period, the methods used for design and analysis have evolved in response to more severe challenges and with a better understanding of the relationship between prediction and actual performance. Effective stress principles have been incorporated into some of these methods. The arguments for including effective stress principles are that the fundamental response of soils is dependent upon effective stresses and not total stresses. Therefore, it is reasonable to anticipate that better, more accurate, predictions of performance can be made using effective stress methods.

The capacity of a pile can be addressed in several major problem groups: the axial capacity in tension or compression, the lateral capacity and behavior of groups. Both static and dynamic behavior are important problems. Static axial capacity is the most important design problem and has been studied much more extensively than all others. For this reason, and because most fundamental principles of pile behavior apply to all the major problem groups, static axial capacity can be used to describe the application of effective stress methods to offshore piles.

All existing rational methods for predicting the axial capacity, Q_c (compression) or Q_t (tension), are based on establishing a relationship between the applied load and resistance along the shaft and at the tip. Thus, in compression,

$$Q_c = Q_{pc} + \int_{surface} \tau(z)dz - W \qquad \text{...(1)}$$

and in tension,

$$Q_t = Q_{pt} + \int_{surface} \tau(z)dz + W \qquad ...(2)$$

where:

Q_{pc} = bearing capacity of the point in compression

Q_{pt} = contribution of point effects in tension

$\tau(z)$ = shearing resistance along the pile shaft

W = weight of the pile (often ignored).

The essence of pile design is to relate soil properties and site characteristics to Q_{pc}, Q_{pt}, and $\tau(z)$. The differences between various pile design methods is primarily a difference in relating soil properties to these three components of the design equations. The significance of effective stress methods applied to pile design also can be illustrated in these terms.

Within the various effective stress methods applied to offshore piles, there are also a few recurring basic questions. Defining Q_{pc}, Q_{pt} or $\tau(z)$ requires a definition of the effective stress state, $\bar{\sigma}_v$ and $\bar{\sigma}_H$, within the soil and the parameters relating the effective stress state to strength or shearing resistance. Pore pressure change is associated with most aspects of pile installation and loading. Consequently, the definition of the effective stress state requires more information than only the hydrostatic groundwater level. The differences in behavior among various soil types, such as normally consolidated clays versus heavily overconsolidated clays, can be related to the differences in excess pore pressure change. Finally, the relationship between effective stress state and strength or shearing resistance can be described in a fundamental way using effective stress parameters. In summary, the basic elements of all effective stress methods applied to piles are:

(a) definition of effective vertical stress

(b) knowledge of the earth stress coefficient relating horizontal to vertical effective stress, $K = \bar{\sigma}_H/\bar{\sigma}_v$

(c) changes in pore pressure and drainage during installation and loading

(d) effective stress soil and interface properties.

In this report, the four items listed above will be discussed in a general sense rather than with reference to a particular design method. Individual sections of the report will then be presented for specific problems.

Historical Overview of Pile Design Methods

Three methods are used to design piles: 1) load tests, 2) pile-driving formulas, or 3) rational predictive equations such as Eqn. 1 or 2. Only the last approach is being considered in this report. McClelland et al. (15) described the state-of-practice for axial compression capacity of piles as:

$$Q_c = f\ A_s + q\ A_p \qquad \text{...(3)}$$

where

A_s = embedded surface area

A_p = pile end area

f = unit skin friction

and q = unit end bearing.

The unit skin function, f, can be described most generally as:

$$f = C_a + \sigma_H \tan\delta \qquad \text{...(4)}$$

where

C_a = adhesion which is independent of normal stress

$\tan\delta$ = coefficient of friction between the soil and the pile.

If f or A_s is variable over the length of the pile, then Eqn. 3 must be written as a summation or with average values.

Typically, the two terms in Eqn. 4 are not operative at the same time. In one group of problems, the "adhesion" or undrained shearing resistance, S_u, is used as the basis for design and friction is assumed to be zero. Conversely, when normal stress and friction are used, as in sands, the adhesion is taken to be zero. The traditional total stress methods for predicting the capacity of a friction pile in clay, therefore, reduce to:

$$Q = f\ A_s = \alpha\ S_u A_s \qquad \text{...(5)}$$

where α equals a coefficient ranging between 0.2 to 1.5, depending upon the pile type, method of installation and type of soil. During the past twenty years, this basic approach has been refined periodically as more data have become available (24,25,28). The recommendation for what value of α should be used is typically based on either the measured shearing resistance of the soil, S_u, or the overconsolidation ratio, OCR.

The α-methods for predicting the axial capacity of a pile exemplify both the advantages and the shortcomings of the total stress approach. The principal advantage is simplicity. A single soil property, the undrained shearing resistance, is needed for design. A second advantage is that actual field performance, in pile test data and case study analysis, is used as the basis for calibration. Consequently, the effects of actual installation and loading procedures are implicitly included in the design method. The disadvantage of the total stress methods is that the various factors controlling pile capacity are not separated and are not addressed specifically. Consequently, extrapolation beyond the conditions of the calibration data set cannot be done rationally.

The most generally used alternative to total stress methods based on undrained shearing resistance is to use a strength defined as a function of effective normal stress and using effective stress parameters, such as:

$$f = \bar{\sigma}_N \tan\delta = K\ \bar{\sigma}_v \tan\delta \qquad \text{...(6)}$$

where, $\bar{\sigma}_N$ = the effective normal stress on the shaft.

This equation is usually redefined following the suggestions of Burland (2), Chandler (4) and Meyerhof (16) to

$$f = \beta\bar{\sigma}_v \qquad ...(7)$$

where, β = effective stress skin friction factor.

Pile design incorporating some form of Eqn. 7 is generally called "effective stress design" and the techniques used are described as the "β methods" of pile design. Values of β have been recommended for a variety of situations and based on several different principles which will be developed in subsequent parts of this report.

Although most rational methods for estimating the capacity or resistance for a pile shaft are either of the total stress (α methods) or effective stress (β methods) type, there are a few others used. One hybrid method proposed by Vijayvergiya and Focht (26) uses both the effective vertical stress and the undrained shearing resistance in a design equation:

$$f = \lambda(\bar{\sigma}_v + 2S_u) \qquad ...(8)$$

Values of λ have been recommended based on analysis of some 47 pile load tests in clay. While successful application of this method in the Gulf of Mexico has been reported, there also have been unfavorable comparisons with predictions and measurements from some large pile load test programs (23). The λ-method is not a true effective stress method and will not be considered in any detail in this report.

Effects of Tip and Shaft Weight

The capacity of an offshore pile is almost always dominated by shaft capacity which will be covered in the remainder to this report. Tip capacity is important in some situations, however. Tip capacity for an offshore pile in compression is usually estimated to be:

$$Q_{pc} = (N_c c + N_q \bar{\sigma}_v)A_t \qquad ...(9)$$

where, N_c and N_q are bearing capacity factors.

In total stress analysis, N_c is usually taken to be 9 and c is S_u. This term is often small for clay soils in comparison with the shaft capacity. In effective stress analysis involving granular soils, the value of the second term may be very large and significant with respect to other terms in Eqn. 1. Appropriate design values for N_q are given, as a function of ϕ', in most references on pile design (9).

The tip capacity for an offshore pile in tension is subject to much more uncertainty than the capacity in compression. The conservative, and often used, limit of capacity is zero. Suction or adhesion effects have been proposed for clay soils under undrained conditions, however, and some pile load tests confirm that a deep pile in clay will develop additional tension capacity as a tip effect. The bearing capacity equation (Eqn. 9) can be used in a modified form to predict the tension effects of a tip. It is common to use $N_c = 7$ (12).

Tip suctions have been measured below piles and drilled shafts in clays subjected to undrained loading. Suction stresses are limited to one atmosphere (101 kN/m^2) and are almost always ignored.

The self-weight of a pile, W in Eqn. 1 or 2, is not ignored. Especially in soft clay soils offshore, the weight of the pile is an important element in evaluating foundation capacity.

EFFECTIVE STRESS SHAFT CAPACITY

Introduction

In this section of the report, the major aspects of effective stress methods for predicting shaft capacity of an offshore pile will be presented in the context of "β factor" design. These aspects have been presented in one or more published reports or references which constitute the body of information about effective stress pile design methods. Most, but not all, of these publications have been developed in the context of the β factor. To provide continuity, and to make this report easier to use, all of the principal aspects of effective stress design are expressed in terms of the β factor even though the original development of the concept might have been done without reference to β. In a few cases, this approach limits the presentation of a fundamental aspect of effective stress design. More detailed discussion is presented, in these situations, to establish the link between the original concept development and the β method.

Drained Conditions

The most direct expression of shaft capacity using effective stresses is:

$$Q_s = \int_0^L A(z)\bar{\sigma}_v(z)K_0 \tan\delta \, dz \qquad \text{...(10)}$$

Defining the effective stress skin friction factor β = Ktanδ given (16):

$$Q_s = \int_0^L A(z)\beta\bar{\sigma}_v(z)dz \qquad \text{...(11)}$$

For application of this design method, there must be an accurate assessment of the in situ effective vertical stress, $\bar{\sigma}_v$. This is usually a simple calculation requiring only a measurement of the distribution of unit weight, γ. If the pore pressure can be defined by observation or measurement, then:

$$\bar{\sigma}_v = \gamma z - u \qquad \text{...(12)}$$

An important and frequent complication offshore is that the pore pressure distribution is not hydrostatic with respect to sea level or some other phreatic surface. Under these conditions the soil profile is described as underconsolidated (20). A more detailed site study and evaluation must be done to define the state of pore pressure in an underconsolidated profile when using effective stress methods for pile design.

The value of β for use in Eqn. 11 has been defined by many different methods, some empirical and some rational. A rational definition of the β factor requires knowledge of both K and δ. The earth pressure coefficient K can range from the active earth pressure coefficient, K_A, through the coefficient of earth pressure at rest, K_0, to the passive earth pressure coefficient, K_p. Under certain conditions, almost any of these values could be applicable to pile design. Consequently, one of the major research questions concerning effective stress methods applied to piles is the selection of the appropriate value of K (see following section).

There is a consensus on several key aspects of the earth pressure coefficient in pile design.

- there is significant change in the value of earth pressure coefficient for some soils and some methods of pile installation, but not others.
- K remains approximately constant with a value near K_0 for piles in contractive soils (normally consolidated or lightly overconsolidated) (5,6,22,23,29)
- K_0 varies significantly with overconsolidation ratio, OCR (14)
- in overconsolidated clays there is a marked decrease in K as the amount of disturbance associated with pile installation and loading increases (5,22)

In the following sections of the report, more specific discussions of K and K_0 are presented.

The remaining term in the basic effective stress design equation is the coefficient of friction between the pile and soil, $\tan\delta$. This factor has been the subject of some laboratory and field research (9,11) supporting recommendation for various soils and pile materials (Table 1). The upper bound of interface capacity is $\tan\delta = \tan\bar{\phi}$ and this may be appropriate for rough concrete piles and similar materials. For smooth concrete and steel, the pile-soil interface friction is lower than $\tan\bar{\phi}$ and this factor must be taken into account in effective stress design.

Table 1: Recommended values for pile-soil interface friction

Soil Type	Rough Concrete Cast in Place	(Timber)	Smooth Concrete, Steel
Clean Sandy Gravel	$\bar{\phi}$[1]	0.65 $\bar{\phi}$	0.60 $\bar{\phi}$
Silty Sandy Gravel	$\bar{\phi}$	0.60 $\bar{\phi}$	0.60 $\bar{\phi}$
Medium Sand	$\bar{\phi}$	0.90 $\bar{\phi}$	0.80 $\bar{\phi}$
Silt	$\bar{\phi}$	0.85 $\bar{\phi}$	0.75 $\bar{\phi}$

[1] $\bar{\phi}$ is measured by direct shear test on remodeled soil specimen, or equivalent.

Recommendations for the "adhesion" to pile surfaces by clay soils have also been proposed (9,25). These adhesion values are presented as a function of the total stress strength - the undrained shearing resistance or "cohesion". This use of shearing resistance is not consistent with effective stress methods, although it is possible to adapt the recommended "cohesion-adhesion" relationships to an effective stress form (Table 1). It is generally accepted that $c' = 0$ for all pile interface strength relationships; consequently, the only soil-pile interface strength characteristic used in effective stress methods is $\tan\delta$.

Coefficient of Earth Pressure at Rest - K_o

The insitu state of effective stress, especially the horizontal effective stress, is a key element in any effective stress pile design method. To provide a basis for this section of the report, it is appropriate to review that state of knowledge, of horizontal earth pressure and the coefficient of earth pressure.

Unfortunately, it is still beyond the state-of-the-art to predict, reliably, the in-situ stress state in soil. Significant advances have been made, primarily in the development of in-situ measuring techniques, which promise a relatively simple approach for the future. However, at the present time, the basis for estimates is normally limited to the stress history of the soil deposit. A typical stress path followed by loading and unloading a soil provides an illustration of these relationships (Fig. 1).

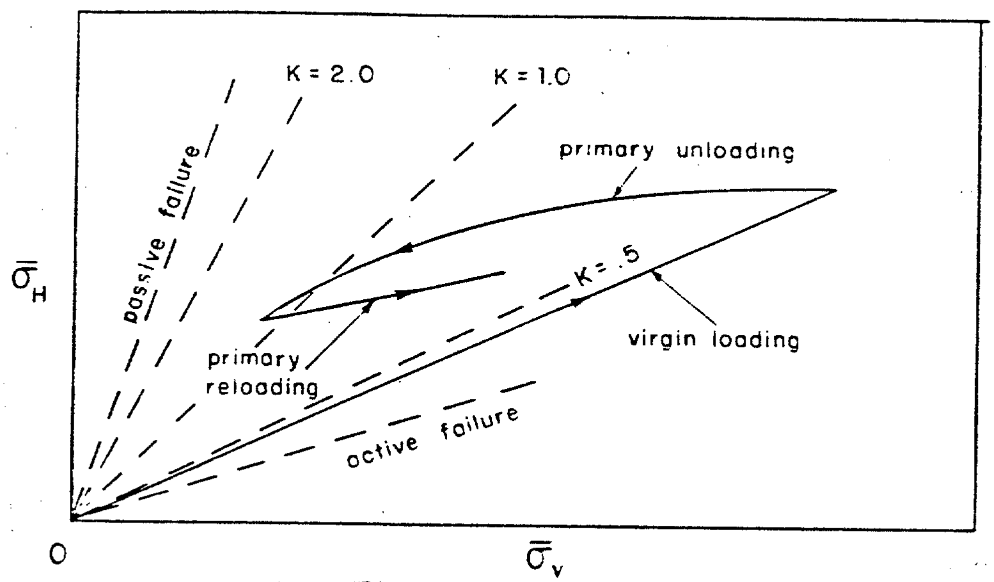


Figure 1: Stress paths illustrating various consolidation histories

In Fig. 1, the relative values of principal stresses (and K_0) are shown to vary for simple stress histories. Stress path 0-B represents virgin loading caused by deposition, surface fill placement, etc. Along this stress path, the soil is normally consolidated and the earth pressure coefficient is K_{onc}. It is accepted that for many soils K_{onc} has a constant value over a wide range of stress. Stress path B-D represents primary unloading caused by erosion, excavation, etc. Along this stress path, the soil is developing increased overconsolidation and the earth pressure coefficient is K_{opu}. Continued unloading beyond D could lead to passive stress failure in the soil. Stress path D-E represents primary reloading caused by renewed deposition, fill placement, etc. Along this stress path, the overconsolidation is being reduced and the earth pressure coefficient is K_{opr}. Continuation along this stress path through B would result in further virgin loading. Examination of these stress paths shows that different K_o values can exist for a given vertical stress. For example, states A, C and E represent the same vertical stress but significantly different horizontal stresses and therefore K_o values.

In addition to the simple stress histories shown, further unloading and reloading may occur. Assuming E as a reference stress state, small increments of stress leading to a reloading-unloading cycle would follow the small stress loop indicated by dotted lines. This could correspond to cyclic loading or disturbance associated with pile installation. Larger stress increments would lead to larger stress loops.

The stress histories illustrated above point to the importance of following the correct stress paths to predict K_o with any degree of confidence. In the following sections, equations are given for predicting K_o during virgin loading, primary unloading and primary reloading. Use of these equations allows the engineer to bound the likely range of in-situ stresses.

Virgin Loading

Numerous proposals have been made during the past four decades for equations to predict K_o during virgin loading (K_{onc}). A recent review of these relationships (14) has confirmed that K_{onc} can be best predicted by the simplified Jaky relationship given below:

$$K_{onc} = 1 - \sin\bar{\phi} \qquad ...(13)$$

The data base used to establish this correlation included some 127 data points on a wide variety of soil materials, with $\bar{\phi}$ varying from 10° to 57°. Other relationships proposed in the literature did not exhibit correlations as good as that of Eqn. 13.

Primary Unloading

The behavior of soils during primary unloading has also received a fair amount of attention during the past one and one-half decades. Although a number of proposals have been made, the equation for K_o during primary unloading (K_{opu}) which is most commonly used is:

$$K_{opu} = K_{onc}\ OCR^{\alpha} \qquad ...(14)$$

in which: OCR = overconsolidation ratio and α = constant. Several proposals have been made for the value of α, but the study by Mayne and Kulhawy (14), encompassing over 90 data points, showed that α can be described by:

$$\alpha = \sin\bar{\phi} \qquad ...(15)$$

Substituting Eqns. 13 and 15 into 14 yields:

$$K_{opu} = (1-\sin\bar{\phi})OCR^{\sin\bar{\phi}} \qquad ...(16)$$

which incorporates only the stress history (OCR) and the effective stress friction angle ($\bar{\phi}$).

Primary Reloading

The behavior of soils during primary reloading has received very little attention in the literature, and the data base is very meager. Mayne and Kulhawy (14) have examined the available data base and have suggested the following. If we take point D on Fig. 1 (maximum OCR) as the reference point, we can assume a linear relationship along stress path D-E which is given as:

$$(\bar{\sigma}_H - \bar{\sigma}_{H_{min}}) = m_r(\bar{\sigma}_v - \bar{\sigma}_{v_{min}}) \qquad ...(17)$$

in which: m_r = constant. Rewriting this equation yields:

$$K_{opr} = K_{onc} \frac{OCR}{OCR_{max}^{(1-\alpha)}} + m_r \left(1 - \frac{OCR}{OCR_{max}}\right) \qquad ...(18)$$

in which: OCR_{max} = OCR at point D in Fig. 1. Examination of the limited data (15 points) suggests that

$$m_r = \frac{3}{4}(1-\sin\bar{\phi}) \qquad ...(19)$$

Substitution of Eqns. 19, 15 and 13 into 18 yields:

$$K_{opr} = (1-\sin\bar{\phi}) \frac{OCR}{OCR_{max}^{(1-\sin\phi')}} + \frac{3}{4}\left(1 - \frac{OCR}{OCR_{max}}\right) \qquad ...(20)$$

This equation includes only the stress history (OCR and OCR_{max}) and $\bar{\phi}$. Although this equation is based on limited data, it is useful as a first approximation at this time. Eqn. 20 is a general equation covering 13 and 16 as specific cases.

Variations in the Earth Pressure During Construction and Loading

The process of installing a pile, either by diving or placement in a prebored hole, will cause significant changes in the lateral stress acting on the pile. These changes may involve changes in pore pressure and subsequent drainage or dissipation of the pore pressures with time. In this situation, there will be changes in the horizontal effective stress, and the pile capacity, with time. The manifestation of these changes are pile "set-up" in contractive soils and the decrease in pile adhesion in dilative soils (24,25). In principle, the changes in pile capacity during installation and loading can be described using effective stress methods and β factor design. Unfortunately, research and data collection applied to this technique have only recently been started. It is reasonable to expect that this methodology will develop in several stages. A first stage will be the description of bounds to the range of β or K associated with various types of soil and pile installation. The next stage will be to collect and interpret data within the range of the bounding solutions. These data will provide an empirical link between theory and the poorly defined aspects of practice.

Some work has already been done to define the bounding values noted above and in the following sections some of these bounding values will be reviewed. The bounding values are typically associated with one of two effects: the changes associated with reconsolidation after the pile is installed and the changes associated with subsequent loading of the pile. It may be more realistic to recognize that the distinction between these two effects is vague and that they could be considered together. Sangrey and Kulhawy (22) took this approach in developing a critical state methodology for evaluating the capacity of drilled shafts. They established bounds based on the known limits of behavior for soils subjected to cyclic loading, both drained and undrained. Using these bounds to define a disturbance factor, DF, ranging from the totally undisturbed limit (DF=0) to the completely disturbed limit (DF=1.0), they were able to describe both installation and loading sequences which increased DF from 0 toward 1.0.

In contrast to this approach, Esrig and Kirby (6) have separated the process of effective stress response of the pile-soil system into a number of discrete segments:

- initial state of stress
- stress changes due to installation
- reconsolidation
- stress changes due to pile loading.

For each of these steps, they propose a behavioral model and the total effect is then the sum of these steps. The methodology of Esrig and Kirby is primarily concerned with predicting undrained shearing resistance and is described in more detail in a following section.

In summary, the changes in response of the soil-pile system during construction and loading can be considered using effective stress methods. Some work has been done to develop methods for doing this, however, most of the information has been concerned with bounding solutions only.

Empirical β-Methods

Effective stress design methods for piles can be calibrated using empirical data rather than theoretical values or limits. The principal contribution of this type comes from the work of Meyerhof (16) who proposed that the effective stress design equation:

$$Q_s = \int_o^L A(z)\beta\bar{\sigma}_v(z)dz \qquad \text{...(21)}$$

could be used in analysis of pile load test data. Fig. 2 is Meyerhof's recommendation for values of β applicable to piles in soft and medium (contractive) clays. Several aspects of this recommendation should be noted. First is the lack of any distinction between pile materials. Since it is generally recognized that the coefficient of friction between soil and piles is different for different pile materials

(11), some of the scatter within the data in Fig. 2 may be a result of this factor. On the other hand, there may be some systematic error in the case records. For example, most long piles are fabricated from steel and would have a low coefficient of friction between pile and soil, $\tan\delta << \tan\bar{\phi}$. This may explain the low values of β for long piles or deep penetration compared with short piles which could be timber or even rough concrete with $\tan\delta = \tan\bar{\phi}$.

A second difficulty with the data shown in Fig. 2 is that the effects of construction and loading, as well as the effects of time, have not been separated. Consequently, piles tested immediately after driving are being compared with piles where there has been sufficient time for set-up. These and similar factors should add to the scatter of data in Fig. 2.

It is noteworthy that the range of β from the empirical studies of Meyerhof does compare favorably with the range predicted using the rational models described earlier. For

$$\beta = K\tan\delta \qquad ...(22)$$

and if, for normally consolidated clays, $K = K_o = 1 - \sin\bar{\phi}$ and assuming that $\tan\delta = \tan\bar{\phi}$, values of β can be calculated, Table 2. The range of these predictions corresponds closely to the upper range of β in Fig. 2. The value of δ for long steel piles would be approximately $\bar{\phi}/2$ and, as indicated in Table 2, the values of β would be lower. This compares favorably with the lower range of data in Fig. 2.

Meyerhof (16) also presents recommended empirical relationships for β applicable to piles in heavily overconsolidated clays. As shown in Fig. 3a and 3b, the values of β are much higher and the scatter of the empirical data is greater. Both of these trends are to be expected, especially since the population of data in Fig. 3a and 3b represents a wide variety of pile materials, construction and loading methods.

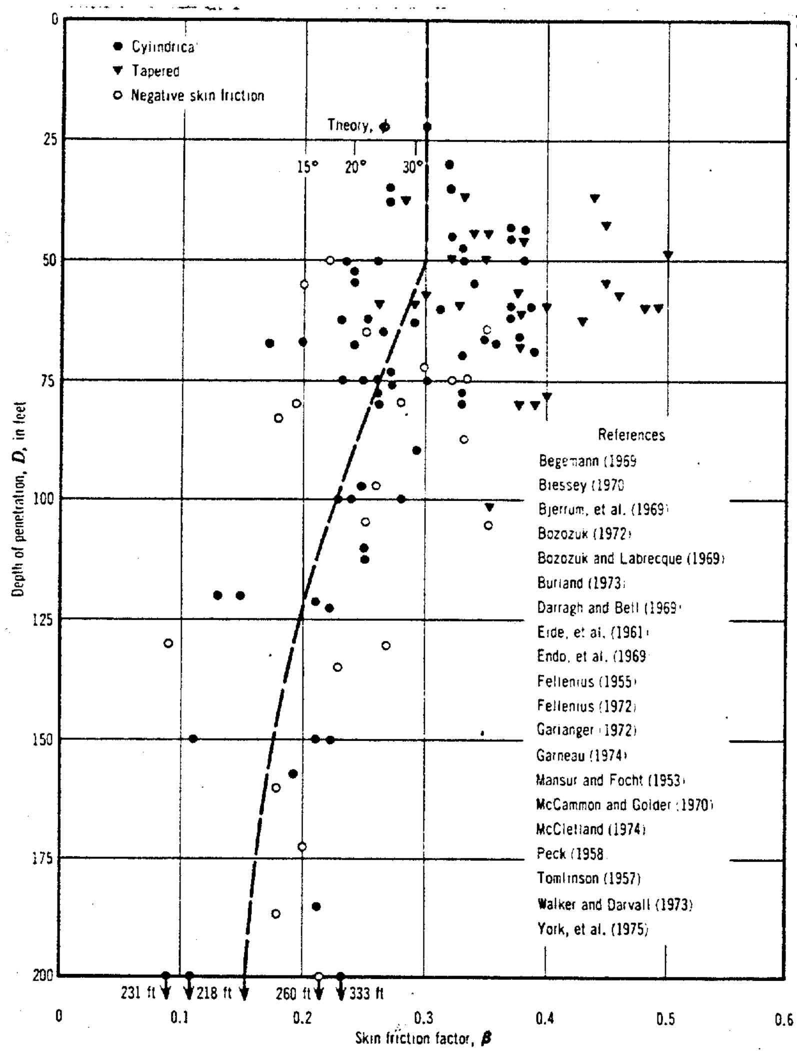
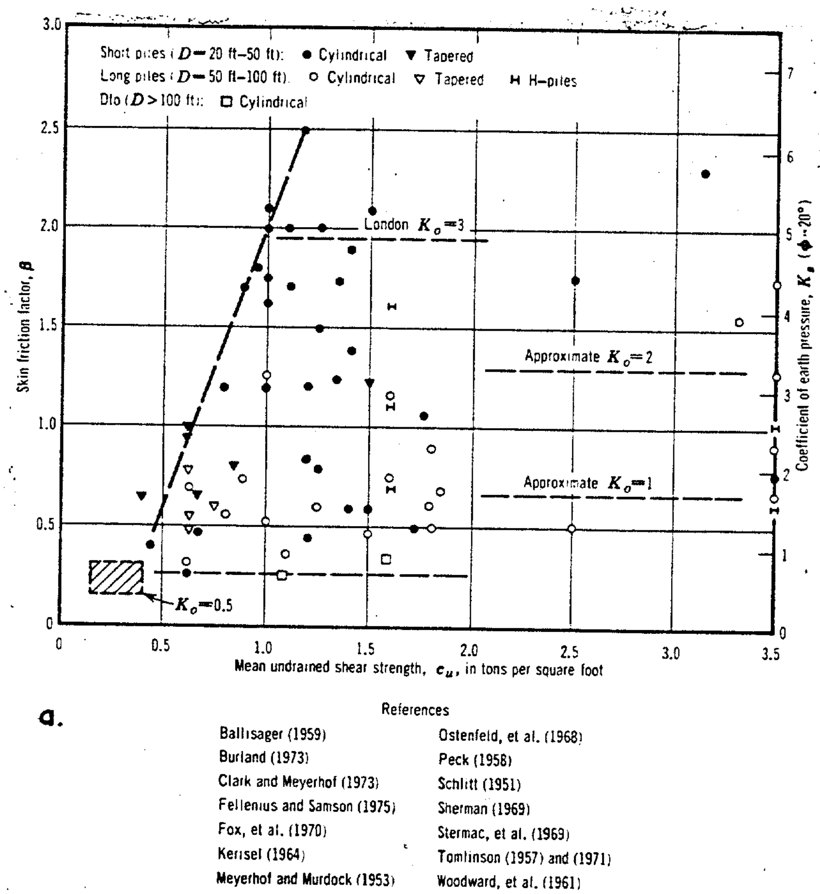


Figure 2: Skin Friction factors for piles driven in soft and medium clays (from Meyerhof, 16)



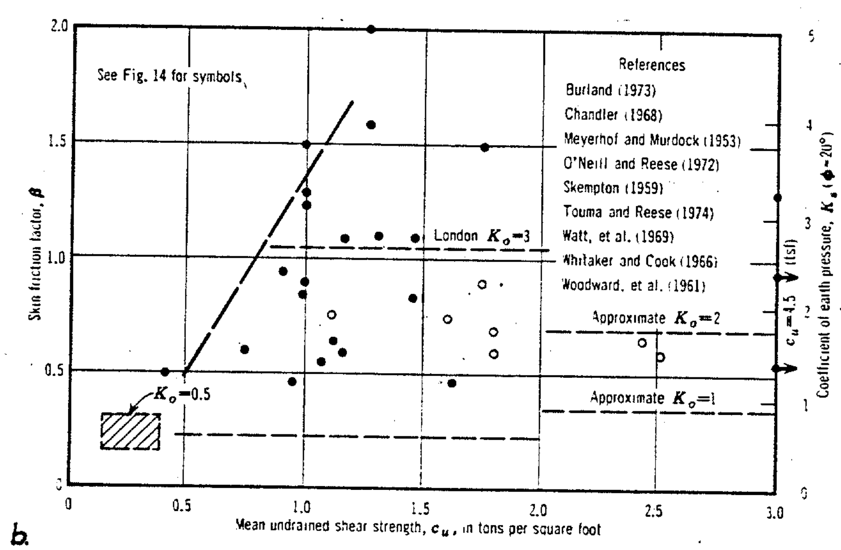


Figure 3: Skin Friction factors for piles a) driven in stiff clay, b) placed in prebored holes in stiff clay (from Meyerhof, 16)

Table 2: Theoretical values of β for normally consolidated soils (Eqn. 21)

$\bar{\phi}'$	β	β for $\sigma=\bar{\phi}'/2$
15°	.20	.10
20°	.24	.12
25°	.27	.13
30°	.29	.13

In summary, empirical analysis of pile load test data using the framework of the effective stress β method has produced recommendations for design. With the addition of more data, and with additional refinement of the analysis, this approach can be extended. Without the refinement of distinguishing among the various factors like construction and loading technique, the uncertainty of empirical relationships is large.

Predicting Undrained Capacity

Effective stress methods for pile design have been proposed using a predicted value of the undrained shearing resistance, S_u, as the basis for design. In this case, the design equation is:

$$Q_s = \int_o^L A(z)S_u(z)dz \qquad \text{...(23)}$$

The predicted undrained shearing resistance need not be restricted to the initial insitu undrained shearing resistance, but can be a prediction of strength after various kinds of disturbance and with time for adjustment of pore pressures. In addition to the empirical methods for relating the undrained shearing resistance to the effective vertical stress described in the previous section of this report, several theoretical and rational methods have been suggested. In all of these methods the undrained shearing resistance is expressed as a variable with depth or effective vertical stress. Therefore, Eqn. 22 can be rewritten as:

$$Q_s = \int_o^L A(z)\beta^*\bar{\sigma}_v(z)dz \qquad ...(24)$$

where β^* is simply a form of β used to describe the relationship of undrained shearing resistance to effective vertical stress

$$S_u(z) = \beta^*\bar{\sigma}_v(z) \qquad ...(25)$$

In an early application of this approach, Sangrey and Sexsmith (23) used a simple model developed in the approach of critical state soil mechanics to predict the $S_u/\bar{\sigma}_v$ ratio, or β^*, for application to offshore piles. Their methodology applied only to contractive soils and provided bounding values to the capacity of an offshore pile. A lower bound to capacity was based on developing the critical state strength for the soil at its initial water content (see Fig. 4). This bound was applicable immediately after driving a pile. The more important contribution from Sangrey and Sexsmith was a definition of an upper bound for shearing resistance which would develop after extensive disturbance and the drainage of the resulting excess pore pressure. They argued that the process of driving an offshore pile through soft soils plus the live loading applied in the offshore environment by wind and waves, was sufficient to cause very extensive disturbance to the surrounding soils.

In Fig. 4 and Fig. 5, the upper and lower bound states proposed by Sangrey and Sexsmith (23) are illustrated. The upper bound state is one where the change in void ratio (water content) resulting from disturbance and drainage is limited by the point where the soil reaches the boundary between contractive and dilative states. This behavior is modeled after the studies of soil response to cyclic loading with drainage reported by France and Sangrey (8) and described in more detail in a following section. The additional assumption of Sangrey and Sexsmith is that the limiting state of effective stress around a pile in contractive soil will be the same as the initial insitu state of stress prior to installation. This assumption, that K remains constant, has been supported by several different field and laboratory studies (5,16,29).

Esrig and Kirby with some other co-authors have presented a series of articles (6,7,10,13) describing the development of a critical state model for offshore pile capacity. Their model also contains bounds to the capacity which are illustrated in Figs. 4 and 5. The lower bound is the same as proposed by Sangrey and Sexsmith (23) but the upper bound is different because it is based on the assumption that the horizontal effective stress around a driven pile in soft clay does not return to the initial value but remains larger than it was originally. As a result, the upper bound capacity predicted by Esrig and Kirby is slightly higher than predicted by Sangrey and Sexsmith. The specific design equations for these two methods are based on the geometry of Fig. 5 and can be found in the references.

Several other effective stress methods have been proposed to predict the changing shearing resistance of soils around piles in soft contractive clays (17,19,29). The major differences between these methods are in the state path followed during reconsolidation of the soil. Consequently, these methods predict slightly different upper bound strengths for contractive soils. These differences are illustrated in Figs. 4 and 5.

There has been some extension of the methods described above to include dilative soils, especially heavily overconsolidated clays. The significant problem with these soils is that the bound approached with increasing time and disturbance is a lower bound to capacity. The work of Tomlinson (24,25) has shown that this lower bound can be a small fraction of the upper bound (initial undrained) capacity. In Fig. 6, a summary of the limiting state for dilative soils proposed by several recent studies (6,19,22) is illustrated. There is much more difference in the predicted bounds for

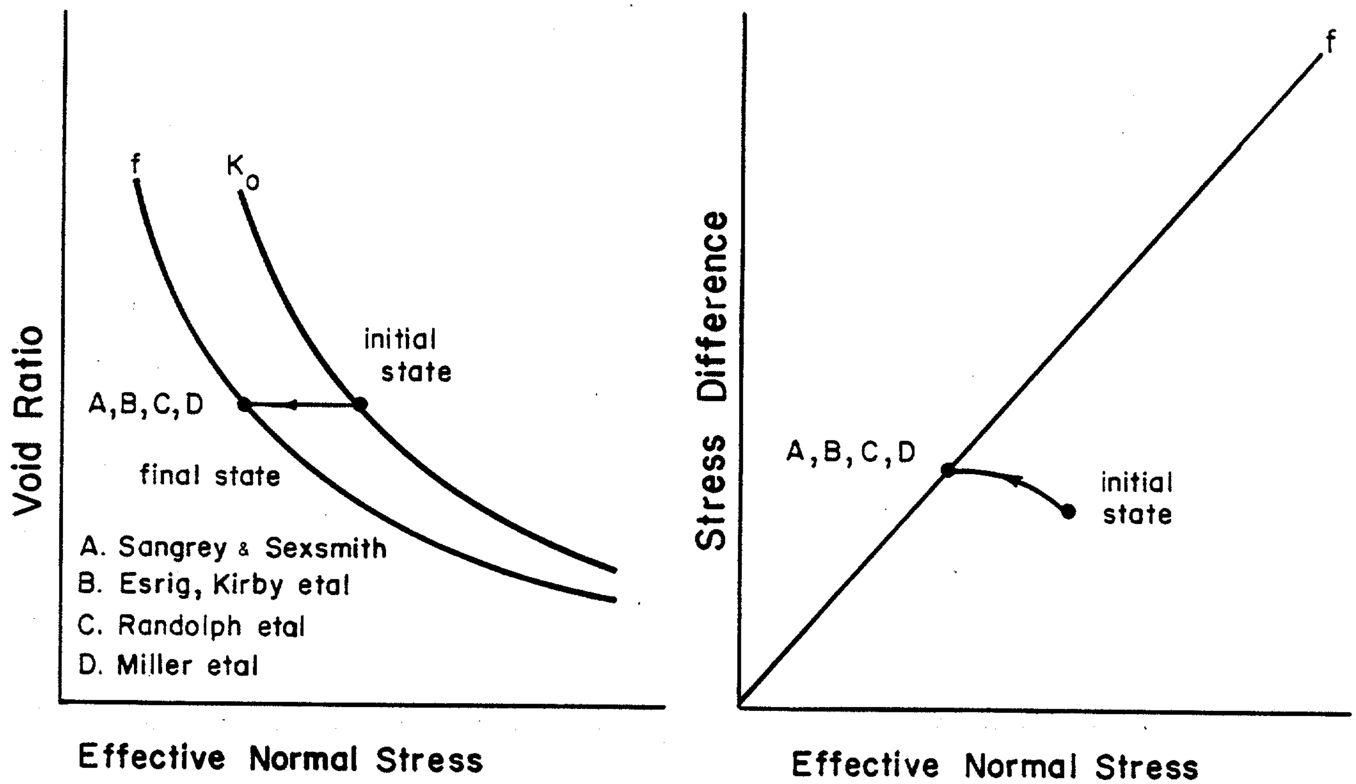


Figure 4: State and stress paths represented by various lower bound estimates of soil strength around piles, contractive soils.

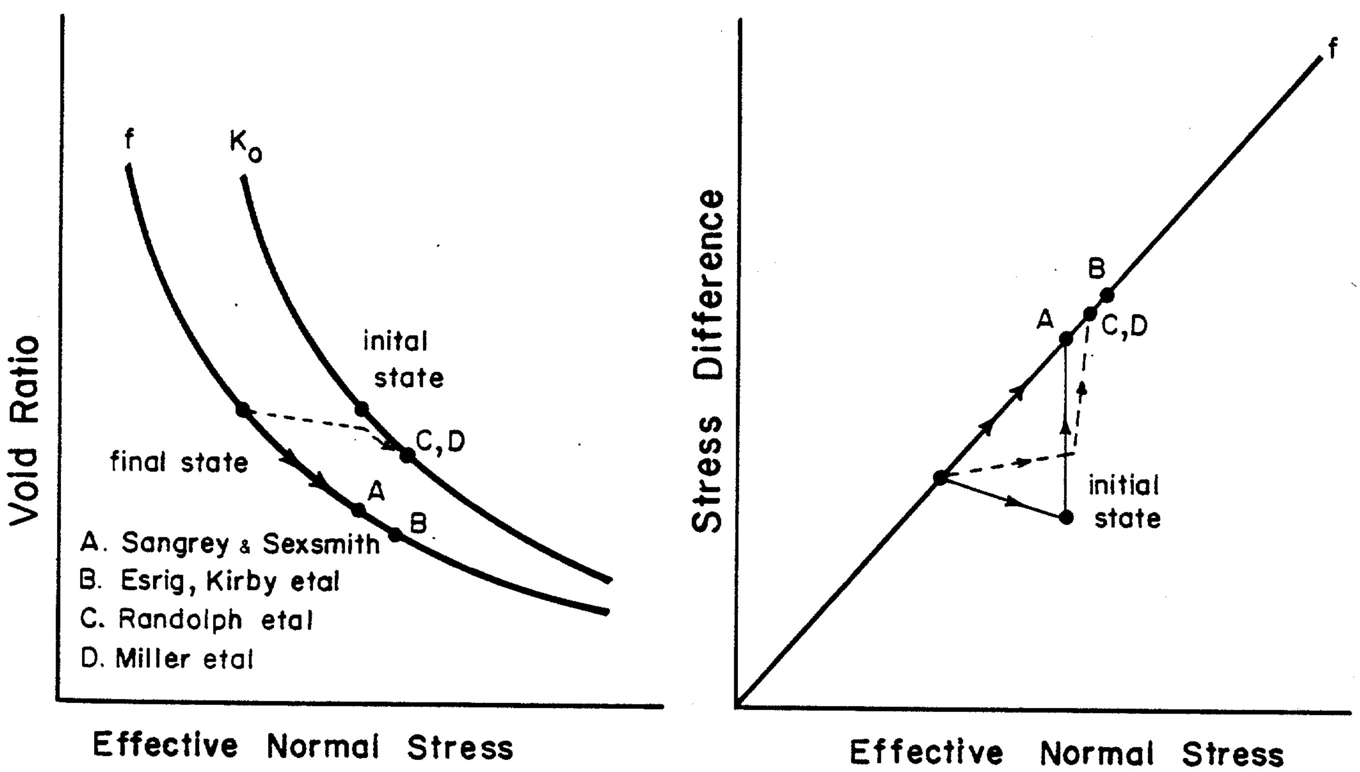


Figure 5: State and stress paths represented by various upper bound estimates of soil strength around piles, contractive soils.

heavily overconsolidated clays than for normally consolidated clays. Limited studies of pile load test data (6) have indicated that the less conservative methods shown in Fig. 6 do clearly overpredict the capacity of piles in overconsolidated clays. It is not clear from studies to date whether the more conservative methods (22) are reasonable estimates or not. Much more work should be done to evaluate all of these effective stress methods using pile load test data along with high quality studies of the soils supporting the piles.

Effects of Cyclic Loading

The live loading of offshore piles is primarily cyclic in nature, reflecting the form of wind and waves, and often represents the dominant loading in a pile design. The driving process itself applies large cyclic loading to soils, if not cyclic loading to failure. Consequently, the fundamental aspects of soil response to cyclic loading, especially cyclic loading with drainage, are applicable to predicting the capacity of piles. The principles of soil response to cyclic loading have also been used to develop bounding values for pile capacity in effective stress design methods. The response to cyclic loading with drainage is also of particular significance in these problems.

In Fig. 7, an illustration of the expected response of clay soil to cyclic loading with drainage is presented. These illustrations are based on well documented field and laboratory behavior (8,25) which demonstrate that, for the contractive soil, the consequence of each interval of undrained loading is a positive residual pore pressure increment which gets smaller as the volume change increases. It has been shown (8) that these volume changes will occur only until the time that the residual pore pressure increment is zero and that this occurs at a point (e_c) corresponding to an intersection with the projected failure line (f). Note that the state of effective stress for this specimen is not at failure but still at the original point, 0_i, in Fig. 7.

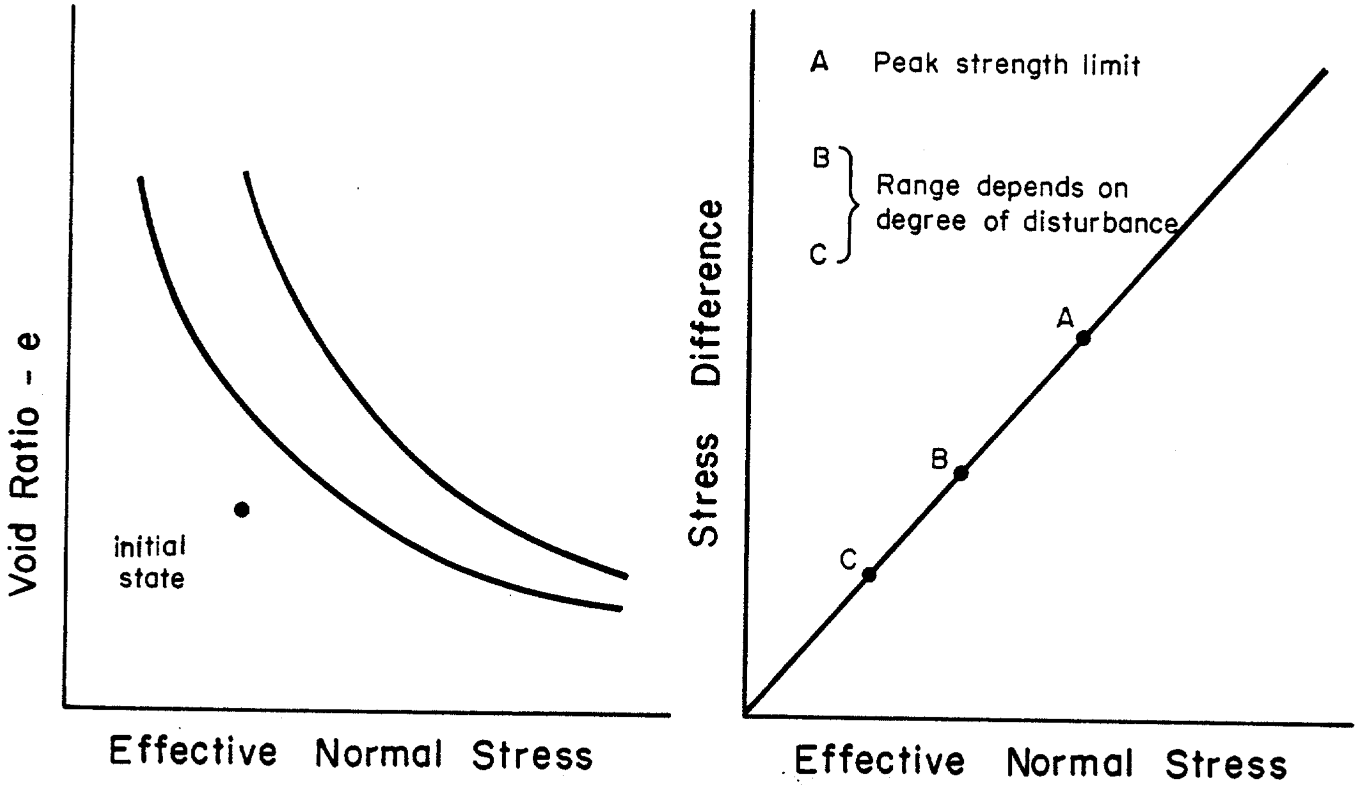


Figure 6: State and stress paths represented by various estimates of soil strength around piles in heavily overconsolidated clays.

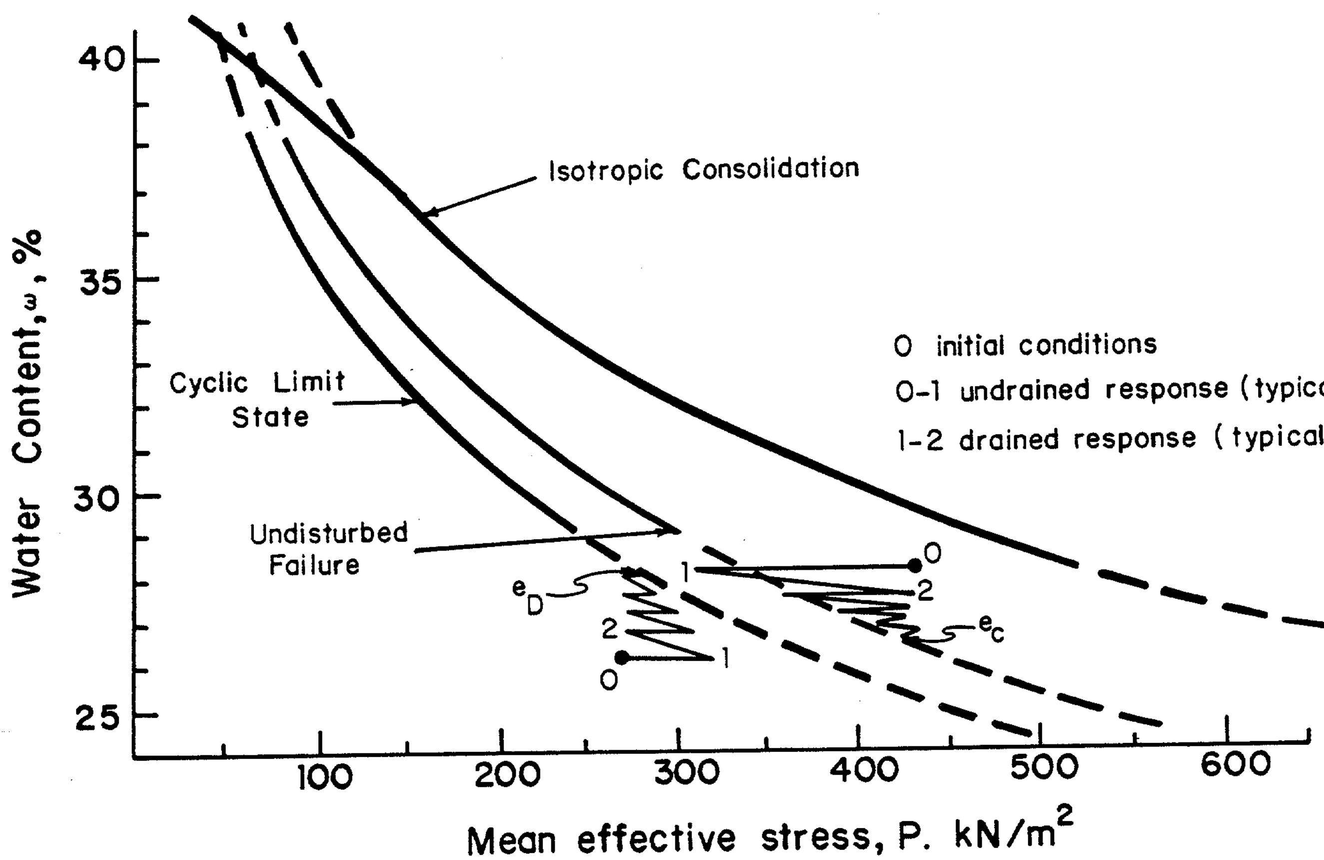


Figure 7: Change in void ratio of soils experiencing cyclic loading with drainage (adapted from France and Sangrey, 8)

In a similar way, the dilative soil will increase in volume under cyclic loading with drainage only until the point, represented by e_D, where the state conditions intersect the failure criterion projection. The strength of heavily overconsolidated clays is generally more complicated than for normally consolidated clays and involving several different limits or criteria. The peak strength (often involving a cohesion intercept), the fully softened strength and the residual strength have all been considered as being applicable to the pile capacity prediction. As illustrated in Fig. 7, and confirmed by testing, the fully softened strength is the operable limit for dilative soils after extensive disturbance by cyclic loading and drainage.

The implications of the previous discussion are very significant, not only for cyclic loading but for other effects as well. Fig. 7 illustrates that there are very specific and predictable limits to the amount of volume change which can occur in a clay soil under the types of conditions associated with installing and loading a pile. At these limits the soils will be at equilibrium under cyclic loading with no potential for change in pore pressure with more cyclic loading.

There will be specific values of undrained shearing resistance associated with these volume change limits and with any of the intermediate points. These values of shearing resistance can be used to predict the capacity of a pile under undrained loading conditions. There has also been an indication that the value of K used in drained loading of the pile will change as a consequence of volume change and softening of dilative soils. These limits and the predictable values of volume change and strength have been incorporated into a recommended effective stress design method for offshore piles subjected to cyclic loading (21). The variation in K with cyclic loading and drainage, which is the key to predicting the long term loading capacity of a pile in heavily overconsolidated clay, has been reported by Sangrey and Kulhawy (22) for application to drilled shafts. Their conclusions can be extended to offshore piles as well.

In summary, the response of offshore piles to cyclic loading has been considered in effective stress design methods. The principal conclusions of these methods is that the changes in pore pressure associated with cyclic loading of piles will lead to drainage. Furthermore, much of the effect of cyclic loading and drainage is expected to occur during installation of the pile. When this is the case, the soil surrounding the pile will have changed by moving toward states of greater equilibrium (e_C or e_D in Fig. 7). At these equilibrium states, the soil will experience no change in pore pressure after cyclic loading. Consequently, at this condition, there is no difference in the capacity of a pile to support static loading or cyclic loading of equal magnitude. This very significant conclusion has been demonstrated by numerous full-scale pile load tests and documented in recommended effective stress design methods (21).

Effects of Very Rapid Rates of Loading on Pile Capacity

The most critical loading condition for most offshore pile foundations is associated with the impact of extreme storm waves. The increase of axial load in this case occurs very rapidly with a rise time which is typically less than 10 seconds. It has been recognized for some time that when soils are loaded very rapidly they offer a different undrained strength from that measured under slower rates of loading (3,27). This behavior is a result of the different levels of excess pore pressures which develop during undrained shear.

A simple illustration of this phenomenon is presented in Fig. 8a. For this undrained test the shearing resistance (S_u), for various rates of loading can be referenced to the strength measured at some arbitrary time to failure, t_r, using the equation:

$$(S_u)_t = (S_u)_r \left[1-\rho\log_{10} \frac{t}{t_r}\right] \qquad \text{...(26)}$$

where

$(S_u)_r$ = the reference strength at time to failure t_r

ρ = $\log_{10}$ change in strength/$(S_u)_r$

Typical data which are available for soils such as those found in the Gulf of Mexico, Fig. 8b, would indicate that the ratio of undrained shearing resistance changes by approximately 10% per log cycle of time from the reference time t_r.

Eqn. 25 can be restated in terms of β factor design methods as:

$$S_{ut} = (\beta^* \bar{\sigma}_v)_r [1 - \rho \log_{10} \frac{t}{t_r}] \qquad \text{...(27)}$$

where $(\beta^* \bar{\sigma}_v)_r$ is used as an alternative way to define the reference undrained shearing resistance (Eqn. 24).

The application of Eqn. 26 to prediction of offshore pile capacity has been described in previous publications (21).

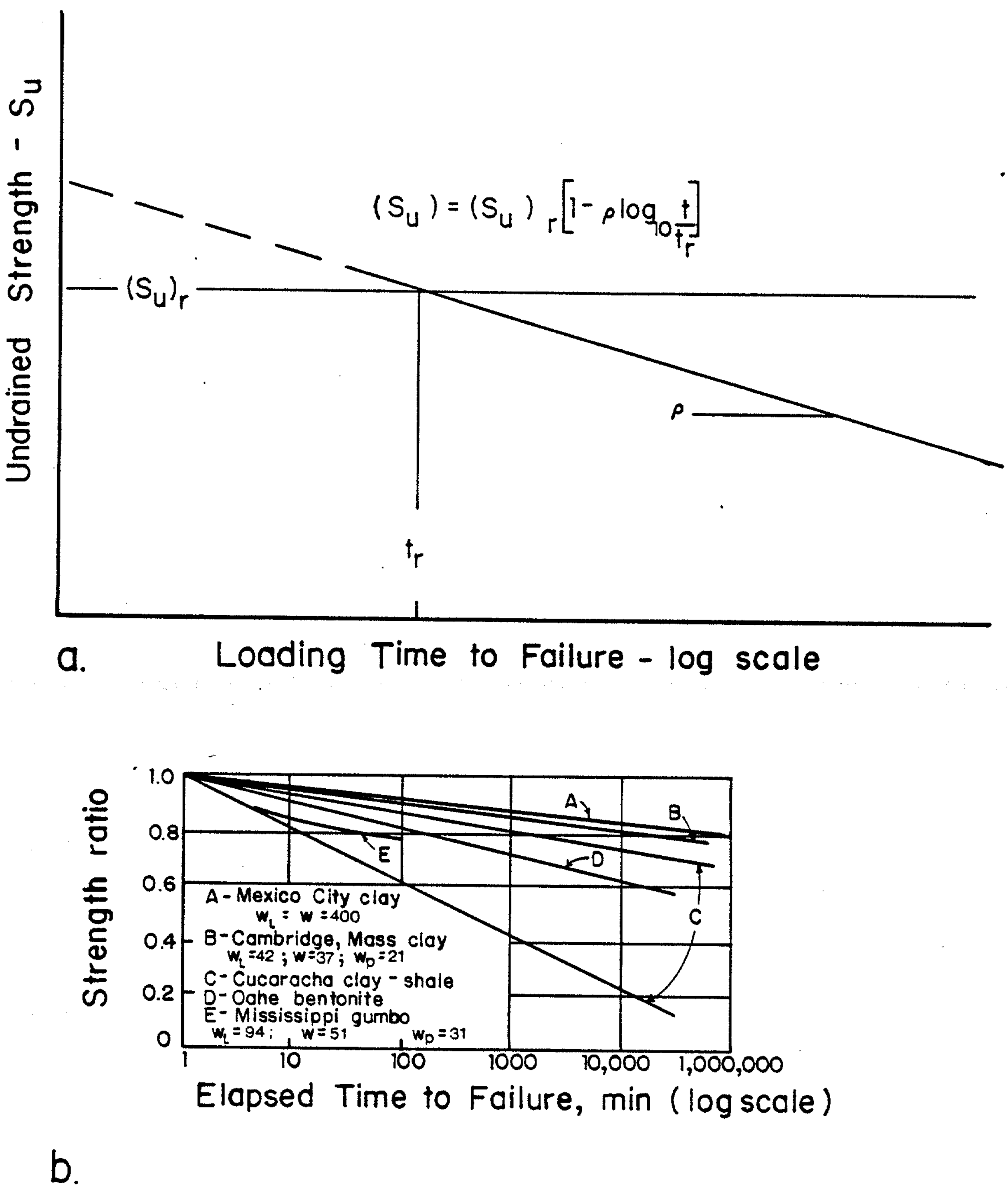


Figure 8: Variation in undrained shearing resistance with rate of loading to failure (adapted from 3 and 23)

SUMMARY AND CONCLUSIONS

In this paper the present state of development of effective stress methods applicable to the design and prediction of performance of offshore piles has been reviewed. The common element of all effective stress design methods is to relate the pile capacity to the shearing resistance as a function of vertical effective stress. Implicit in these methods is the hypothesis that the vertical effective stress can be defined with less uncertainty than other soil or site properties. A variety of effective stress methods has evolved, most of them distinguished by the particular strength characteristic or strength limit they propose to model. Methods based on modeling the drained strength are commonly expressed in the form:

$$\beta = K\tan\delta \qquad \text{...(28)}$$

and the principal concern for design is selecting the appropriate value of K. The initial insitu value of $K=K_o$ can be estimated or can be measured, however, the changes in K associated with installing and loading an offshore pile are not well understood. For contractive soils, there is support for the assumption that K does not change appreciably with pile installation and loading. For dilative, heavily overconsolidated clays on the other hand, there are dramatic changes in K.

Methods based on modeling the undrained shearing resistance of soils can be expressed in the form:

$$\beta^* = \frac{S_u}{\bar{\sigma}_v} \qquad \text{...(29)}$$

Undrained shearing resistance is usually predicted as limiting or bounding values. The undrained methods have been developed for both contractive and dilative soils and for a range of limits from the original capacity of a freshly driven pile to the limit of capacity after extensive disturbance and volume change in the soil surrounding the pile. Some bounding values, especially for soft soils, have been verified by load tests and performance monitoring. The capacity of piles in heavily overconsolidated clays is subject to much greater uncertainty.

The response of offshore piles to cyclic and dynamic loading is an important design consideration. Effective stress methods have been developed for this group of problems and have been applied successfully. These methods can also be expressed in a β factor format.

An early recommendation for effective stress design methods for piles (16) proposed simply that pile load test data be interpreted in terms of a skin resistance factor, β, related to the effective vertical stress

$$f = \beta\bar{\sigma}_v \qquad \text{...(30)}$$

This method has been used successfully to interpret empirical data from both onshore and offshore. A difficulty with this approach as applied to date has been the grouping of load test information without regard for significant factors such as pile material, construction and loading history. Improvement in empirical β factor curves can be anticipated if more well-documented case studies are reported so that these significant factors can be isolated.

There are many aspects of offshore pile design which could be described within the context of effective stress methods but have not been covered in detail in this report because they are not appreciably different in either total or effective stress approaches. Pile-soil interface characteristics, tip capacity, lateral loading and group action fall within this area. Another group of important contributions concerns the problems of stress distribution around piles and the stress distribution when piles are loaded. Important contributions such as those from Parry and Swain (18) have been incorporated into some of the effective stress design methods reviewed in this report, however, a detailed discussion of this factor is considered tangential to the principal objective of the report.

Effective stress design methods for offshore piles are just emerging as both viable alternatives to more traditional methods and also the preferred method in many cases. The pile foundations for some of the largest offshore structures built within the past few years have been designed using effective stress methods. Future development of these methods and confirmation through performance monitoring can only increase the rate at which effective stress methods are adopted.

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